Risk Reduction at Ash Disposal Area No. 2
A Case Study
TVA Johnsonville Fossil Plant
New Johnsonville, Humphreys County, Tennessee

Stephen H. Bickel, PE¹ and Roberto L. Sanchez, PE²

¹Stantec Consulting Services Inc., 1901 Nelson Miller Parkway., Louisville, KY 40223; ²Tennessee Valley Authority, Fossil Power Group, CCP Projects & Engineering, 1101 Market Street, Chattanooga, TN 37402

KEYWORDS: coal ash, fly ash, seepage, slope stability, spillways, hydrology and hydraulics, grouting pipes

INTRODUCTION

Immediately following the December 2008 Kingston dredge cell failure, TVA commissioned Stantec Consulting Services Inc. to conduct Phase 1 Assessments of all CCP disposal facilities at TVA’s eleven fossil plants. These studies were performed to: 1) identify conditions that could affect safety, stability, and/or functionality; 2) determine whether a compelling need for short term corrective actions exists; and 3) prioritize Phase 2 studies and remedial construction projects.

The Phase 1 Assessment for Ash Disposal Area No. 2 at TVA’s Johnsonville plant resulted in a high priority ranking for risk reduction actions. The assessment uncovered issues of concern which included: 1) a lack of adequate hydraulic freeboard; 2) seepage areas which had reportedly increased in size; 3) tall spillway risers that leaned and consisted of stacked concrete pipe sections; 4) submerged spillway outlet pipes that regularly surged and had a history of sinkholes forming above them; and 5) steep exterior dike slopes which likely had less-than-adequate factors of safety for global slope stability.

In early 2009, even before the Phase 1 Assessment final report was complete, TVA began the risk reduction program at Ash Disposal Area No. 2 to address each identified issue. The program began with two projects. The first installed a pipe system that collected seepage in the most critical seepage area and allowed it to be monitored for changes. The second project installed new spillways, lowered the operating pool levels, and increased the hydraulic freeboard.

While these activities ensued, a comprehensive geotechnical exploration was planned and carried out. Additionally, four risk reduction projects were designed, and
construction will be complete on all by June, 2011. Finally, operational changes were made to further decrease risk.

This paper presents the risk reduction program followed by TVA at Ash Disposal Area No. 2. Beginning in early 2009, new spillways were built and the pool level was lowered. Once completed, the old spillways were closed. Slope stability and seepage issues were addressed by rerouting the sluice channel, dewatering, installing graded filters, and buttressing or flattening dike slopes. These efforts have transpired while maintaining the facility’s active status. The goals are: completion by mid-2011 and for Ash Disposal Area No. 2 to achieve the Tennessee Safe Dam Program requirements for a “significant hazard structure.”

JOHNSONVILLE FOSSIL PLANT

Location: The Johnsonville Fossil Plant (JOF), formerly known as the Johnsonville Steam Plant, is located on a 748-acre reservation owned by TVA in west-central Tennessee. The plant site is in the community of New Johnsonville, which is in Humphreys County. It is on the east bank of the Kentucky Lake reservoir, which was created on the Tennessee River in the mid-1940s. The plant is located approximately 12 miles west of Waverly, Tennessee, and approximately 65 miles west of Nashville, Tennessee. Figure 1 provides a general vicinity map for JOF.

![Figure 1: JOF General Vicinity Map](image-url)
**General Information:** The Johnsonville plant contains ten coal-fired steam turbo-generating units. JOF was the second steam-electric project to be built by TVA, and it is currently the oldest fossil plant in the TVA system. Construction at JOF began in May 1949, and the first unit began supplying power in October 1951. Five additional units were added by February 1953, and the last four units were completed in late 1959. Upon its completion JOF was the third largest plant in the TVA system and it was among the ten largest electric generating plants in the world. Currently, the winter net dependable generating capacity for the ten units is approximately 1,250 megawatts.

The ten units at the plant consume a low-sulfur coal blend at a rate of approximately 3.5 million tons per year. This consumption results in a coal combustion products (CCPs) production of about 280,000 tons fly ash and 60,000 tons bottom ash per year (values are given as dry tons). Within the powerhouse, two separate ash handling systems for removing fly and bottom ash were installed. The systems employ high velocity air, water jets, hoppers and pumps, and create slurries that are pumped to the disposal area at velocities high enough to prevent settling within the sluice piping. The plant inputs to the disposal area are approximately 35 million gallons per day (3,250 gpm) when all ten units are operating.

**ASH DISPOSAL AREA NO. 2**

**Description:** Ash Disposal Area No. 2 is the second disposal area constructed at JOF. It was first shown on TVA drawings in 1962, cost estimates were prepared in 1966, and its construction period occurred from 1968 through 1970. Its location is one of three sites identified for the disposal of ash during the original planning for the plant layout in
the 1940s. Ash Disposal Area No. 2 was designed to have a capacity of 4.5 million cubic yards of storage, which equates to about 15 years.

Referring to Figure 3, the site is on a 125-acre constructed island centered approximately 2,000 feet west of the JOF powerhouse. Access onto the island is via a 1,000 foot causeway embankment. The causeway contains a one-lane asphalt access road, discharge piping from the plant, and underground electric utility lines. The island is surrounded by Kentucky Lake to the west and two dredged channels (the Boat Harbor and Condenser Water Inlet channels) to the east.

Ash Disposal Area No. 2 is 87 acres in area, as measured within the dike. The dike crest is a gravel paved road that is approximately two miles in total length. The crest is at Elevation 390 feet msl, or about 30 feet higher than Kentucky Lake summer pool. The dike is from 25 to 30 feet in height, and was constructed with outslopes that varied from 1.5H:1V on the east side to 2.5H:1V along the west side. The slopes are maintained with grass cover and several stands of mature trees are just above the lake edge, around the south and southwest dikes. The old Tennessee River channel is about 200 feet west of the shoreline, and there is a 3,500-foot long riprap zone along the west dike to provide erosion protection against the currents and waves.

Conditions and operations inside the ash disposal area can be separated into three distinct regions. The north region is filled with ash, graded almost level, and maintained in a dewatered state. Surface ditches immediately inside of the clay dike have been established at flat grade from 10 to 15 feet below the dike crest. During wet weather the ditches are dewatered by pumping.

The central region is where the ash handling operations occur. The sluice pipes from the plant terminate at the west end of the causeway and flows are routed through two sluice channels. TVA's ash handling contractor uses long-reach excavators to dip the ash from the sluice channels and deposit the ash on adjacent higher benches where it dewatered. The ash accumulates in these piles throughout the winter and spring. During the summer and fall, the moisture within the ash is reduced, and it is loaded into trucks and hauled to an offsite, permitted landfill in Camden, Tennessee, which is about five miles to the west.

The southern region contains an ash pond complex consisting of three ponds that are separated by ash divider dikes. Denoted as Ponds A, B and C (the stilling pond), they total for about 26 acres of surface area and contain approximately 300 acre-feet in water storage volume. Since some of the ash is not captured by the Contractor's mechanical dipping, the ponds must be periodically dredged in order to maintain the permitted free-water volume required by the Tennessee Department of Environment and Conservation (TDEC).

Construction History: While the formal construction period to create Ash Disposal Area No. 2 occurred from mid-1968 through mid-1970, an initial phase actually took place during the original plant construction. The dredging for the Boat Harbor and Condenser Water Inlet channels resulted in placing hydraulic fill to form the Boat Harbor dike on the
Figure 3: Site Layout for Ash Disposal Area No. 2

west side of the Boat Harbor channel, and the Sangravl dike, located west of the Condenser Water Inlet channel. The hydraulically-placed material would later become portions of Ash Disposal Area No. 2's northeast and southeast dikes, respectively. The dredged materials consisted primarily of alluvial clay and silt, with sandy and gravelly zones intermixed throughout.
It is also noted that a chert layer was encountered approximately four feet above grade during the dredging operation in the Boat Harbor channel. The hard, resistant chert was reportedly dug loose using a mechanical dipper dredge, then picked up and moved into the Boat Harbor dike using the suction dredge. This created a substantial zone of clayey gravel in what would become the south end of the ash disposal area’s northeast dike. Figure 4 shows the 16-inch TVA suction dredge “John S. Scott” working in the channel. During this period, the Boat Harbor dike was built to Elevation 377 feet msl, and the Sangravl dike to Elevation 363 feet msl. The Boat Harbor dike was built higher to provide a breakwater with better protection against waves for the coal barges moored within the coal unloading area.

![Figure 4: TVA 16-inch Suction Dredge John S. Scott Working in Boat Harbor Channel](image)

Beginning in June 1968 and continuing for approximately two years, the remainder of the dike system was constructed. The dike was built from each end of the Boat Harbor dike until it completely enclosed the new ash disposal site. During this period the dike crest was built to Elevation 378 feet msl. This elevation was chosen to place the crest well above potential high water during flooding on the Tennessee River/Kentucky Lake reservoir. Hydraulic fill was used in the lower portion of the dike (up to Elevation 368 to 370 feet), and compacted clay fill was used above. Material for the hydraulic operation was mostly low plastic silty clay dredged from within the disposal area. Clay used to construct above the hydraulic fill was brought from a borrow site located east of the coal stockpile (the old plant construction camp).

During this period the spillway system was also built. It consisted of two sets of three spillway pipes. One set was built in the west dike near the south end (South Spillways), and the second in the west dike near the north end (North Spillways). Each spillway outlet was a 36-inch diameter precast reinforced concrete pipe (RCP) and they were installed in original ground line using the “induced trench” method to reduce loading. A precast concrete junction box was constructed at the inlet end and 48-inch RCP sections were stacked on top of the box to form the spillway riser. Headwalls were not constructed at the outlet pipe ends.
Last, the ash sluice and other pipes from the plant were redirected across the causeway to discharge into the new disposal area. In late spring or summer 1970, Ash Disposal Area No. 2 was complete and ash sluicing from the plant was directed here.

Eight years later in 1978, a third construction phase began at Ash Disposal Area No. 2. At this time the disposal area was reaching its capacity, and the enclosing dike was raised to Elevation 390 feet msl. The dike was raised using the upstream method of dam construction. Initially, a 4-foot thick base of compacted bottom ash was prepared beneath the upstream portion of the new dike, and then compacted clay was used to construct the new dike. The clay was obtained from a borrow site located east of the 500 kV switchyard and from an area on the TVA reservation known as the South Rail Loop area.

In conjunction with this construction phase, a third set of three spillways (East Spillways) was added in the northern end of the southeast dike. These were similar to the two original sets except that anti-seep collars were added, rubber-O-ring gaskets were included at pipe joints, and the "induced trench" of construction was omitted. Finally, a Metals Waste Cleaning Pond (aka: the Chemical Treatment Pond) was constructed on the island immediately outside of Ash Disposal Area No. 2.

The work performed during 1978 and 1979, represented the final phase of construction at Ash Disposal Area No. 2. With the exception of maintenance, repairs, and changes in pool levels or ash handling operations, this TVA facility has remained essentially unchanged for more than 30 years. Instead of an ash disposal area, it became an ash transfer facility where CCPs were temporarily stored prior to being moved to the point of final disposal.

In the early 1980’s, TVA had constructed a third ash disposal site on the JOF reservation called the Railroad Loop Ash Disposal area. Ash and sluice water continued to be pumped into Ash Disposal Area No. 2, but ash deposited there was periodically dredged and pumped over to the 125 acre Railroad Loop area. A forth ash disposal site was also built in an area east from the combustion turbine units. This 35-acre site was referred to the DuPont Dredge cell. These two disposal sites remained active through the 1990’s, and were both closed in the 2002. A third disposal facility in the North Rail Loop area was designed and permitted; however, it was never constructed. By late 2002, the ash handling operation changed to mechanical dewatering at Ash Disposal Area No. 2 and later, the dewatered ash began being trucked to a permitted landfill in Camden.

PHASE 1 ASSESSMENT AND FINDINGS

Scope of Phase 1 Assessment: The Phase 1 Assessment began the second week in January, 2009 (less than three weeks following the Kingston Dredge Cell failure), and it was completed in May. The conclusions and recommendations for Ash Disposal Area No. 2 are included in: “Report of Phase 1 Facility Assessment for Coal Combustion Product Impoundments and Disposal Facilities for Tennessee" dated June 24, 2009. The following is a summary of the assessment.
Three separate site visits were made by the review team during the Phase 1 reconnaissance. The initial visit on January 12, 2009 was a quick, one-day review intended to ascertain whether “imminent and compelling” issues or concerns were noted that could result in another “Kingston-type” failure. The second visit involved a detailed three-day inspection. This visit was scheduled for February 23-25 of 2009 to follow the document reviews. During this visit, issues were noted, located, and photographed. Dike slopes, crest widths, height, freeboard, and other pertinent dimensions were surveyed. Last, a third visit was scheduled during March 2009 when a geotechnical consultant team, retained by the Tennessee Department for Environment and Conservation to evaluate TVA impoundments in Tennessee, made a visit to JOF.

Early in the Phase 1 assessment, TVA made available for review, every known document in its archives pertaining to ash disposal facilities at JOF. These documents included old maps, aerial photographs, correspondence, calculations, geotechnical reports, and drawings that were developed during the planning, design, and construction phases. They also included quarterly and annual dike inspection reports developed over the operational life of the facility. These documents were reviewed by the JOF engineering review team and potential issues were noted for further investigation and/or field review. A total of almost 900 separate documents were reviewed during the Phase 2 assessment. One notable finding was that there were no construction record drawings, construction inspection reports, or quality control test results, contained in the archived materials for Ash Disposal Area No. 2.

In addition to the site reconnaissance and document reviews, interviews were held with several TVA engineers and environmental staff at the plant who were knowledgeable of Ash Disposal Area No. 2. Due to its age, it was not possible to interview any person who had been involved in the initial planning, design or construction.

Phase 1 Findings: The notable issues and concerns found during the Phase 1 Assessment are as follows:

- The ash pond complex operates at a relatively high water level and displayed a freeboard of less than 2 feet. This creates a concern for possible overtopping.

- Steep slopes and hummocky, uneven ground surfaces on northeast and southeast dikes, create slope stability concerns. A “bulge” developed near the lower slope at the south end of the northeast dike during the mid-1990’s and a shallow surface slough occurred following heavy rainfall in the same area during 2002. TVA’s inspection team theorized that the slough may also have been aggravated by the ash dipping in that area.
Figure 5: Inadequate Freeboard Next to Northeast Dike

Figure 6: Steep Slopes and “Bulge” on Northeast Dike
• Use of hydraulic fills, bottom ash and waste materials from electrostatic precipitator construction, and documented seepage areas on the northeast and southeast dikes create concerns for piping, internal erosion, and the general condition of the dike. Beginning in 2002, TVA’s inspections indicated the most significant seepage area (Seep 3A), which is at the toe of the southeast dike approximately 1000 feet south from the causeway, had increased in size. Another TVA inspection in 2008 reported a similar observation. During the Phase 1 site reconnaissance this seep was measured over a length of about 150 along the lower bench. The flow was discernable but could not be quantified.

• The spillway risers are vertically stacked concrete pipe sections, 35 feet in height, and are laterally supported by settled ash only (they are not structurally connected, but held together by gravity). Several risers display a slight tilt that may have been caused by impacts during a dredging operation. This created a concern for structural stability and potential for loss of pool, or complete dike breach should the riser topple or separate at depth.

![Figure 7: Spillway Risers in Ash Pond Complex](image)

• During normal operations the ends of the spillway outlet pipes were submerged and the hydraulics of the riser/outlet pipe combination cause a periodic air discharge (see Figure 6), which indicated a constant change of internal pressure.
Additionally, the pipes were not constructed using restrained joints or gaskets. According to reports, the pipes experienced joint separations that resulted in internal erosion of downstream dike material and formation of sinkholes in the fall of 1992 and spring of 1993. Up to 20 feet in diameter and 15 feet deep, the sinkholes created concern about the integrity of the dike and whether the repairs (slip lining) adequately addressed issue of internal erosion. It was noted that one of the pipes could not be slip-lined for an unknown reason.

There were other observations and concerns that developed from the Phase 1 study, but these were secondary and considered programmatic issues related to a need for improved maintenance and construction documentation.

The information gained during Phase 1 site visits and document reviews provided the basis of knowledge needed to develop recommendations for Phase 2 studies and for initial risk reduction projects to address the operating pool level and observed seepage. Prior to completing the Phase 1 Report, TVA acted on the primary issues of

![Figure 8: Air Surging at End of Outlet Pipes](image)

concern which developed during the Phase 1 process. Three additional information gathering and risk reduction actions began moving forward in late January 2009:

1. TVA authorized the design and installation of the seepage collection system at Seep 3A. Its purpose was to collect seepage at a concentrated point so that it
could be monitored to determine if flows were indeed changing as recent inspection reports seemed to indicate. It also provided an opportunity to observe the nature of the material exposed in the shallow collection system trenches.

2. Recognizing an immediate need to determine the subsurface conditions and slope stability, TVA authorized the start of a comprehensive geotechnical exploration and slope stability evaluation of Ash Disposal Area No. 2.

3. Realizing the existing freeboard was inadequate, and serious concerns were associated with the existing spillway system, TVA authorized design and construction of a new spillway system and subsequent abandonment of the nine old spillway pipes that penetrate the dikes in Ash Disposal Area No. 2.

SEEPAGE COLLECTION AT SEEP 3A

During February, 2009 a collection system was designed and constructed on the lower bench of the Southeast dike at Seep 3A. It consisted of shallow interconnected trenches, each with a perforated pipe encased within clean, free-draining crushed stone. At the downstream end the crushed stone encasement was replaced with compacted clay so that all seepage would be contained inside the pipe and exit at a single outfall. Figure 5 provides a view of the completed system.

The material encountered during its installation was a heterogeneous mixture of clay, and gravel-sized pieces of chert and sand. Due to the amount of concentrated sand and gravel, it is believed that this location was a primary discharge point for the suction dredge during the original the Intake Channel dredging.

A one-liter glass beaker and stopwatch was used to measure flow rates. Samples collected were also visually examined for changes in turbidity. After two months of monitoring, the dry-weather base flow was determined to be about 2.4 gpm (3,500 gpd). The collected water samples were clear and judged to be free from soil particles.

It is noted TVA considered that the data to be provided by the seepage collection system would be essential for decisions involving safety and prioritizing risk reduction actions. Therefore, this work was authorized on an emergency basis. Environmental permitting and processes normally followed when a new pipe outfall is created at a TVA facility was by-passed in this special case. The collection system remained in-place until October, 2010 when this area was remediated during construction of the Southeast Dike Slope stability Improvements Project (to be discussed later).
GEOTECHNICAL EXPLORATION AND SLOPE EVALUATION

Scope of Geotechnical Exploration: The Work Plan for this study was completed and approved in mid-February 2009. Immediately thereafter, three drill rigs and crews were mobilized to JOF. The crews worked continuously and the drilling and sampling work was completed in a six week period. TVA Management was kept informed of findings throughout the investigation. Protocols and guidelines established by the US Army Corps of Engineers were followed. In summary, the geotechnical exploration and analyses were performed in the following manner.

Fourteen separate cross-sections were located around the dike with the average distance between each cross section being about 800 feet. Each cross-section was surveyed to accurately determine the ground line geometry and elevations. At each cross-section a boring was drilled on the dike crest, and a second was drilled at the toe. These borings provide primary information for the seepage and stability analyses. Borings advanced on the dike crest were drilled to 60 feet total depth. Borings drilled at the dike toe were drilled to 50 feet in depth. These depths provided a total depth soil profile of about 80 feet to be developed, or about 2.5 times the dike height, which was judged to be sufficient for analyses. Continuous sampling was performed within the dike and into foundation soils. Penetration test samples were performed to obtained qualitative date regarding subsurface materials. After reviews of this data, offset borings were drilled and undisturbed samples were obtained at targeted depth intervals for shear strength testing. Piezometers were also installed with screens positioned at
targeted depths to provide data for seepage analyses. In addition, twenty soil test borings were advanced for supplemental geotechnical data. These borings averaged about 30 feet in depth. Figure 6 depicts a drill rig working on the dike.

![Drill Rig on Ash Disposal Area No. 2 Dike](image)

**Figure 10: Drill Rig on Ash Disposal Area No. 2 Dike**

During the Phase 1 document review, six reports describing previous geotechnical studies at Ash Disposal Area No. 2 were discovered. From these reports a total of sixty-eight boring logs were reviewed, and a majority of the data contained on the logs was judged to be sufficiently accurate to be used in this geotechnical evaluation. Several of these boring logs depicted data that did not appear reasonable or consistent with information obtained during this study; consequently these logs were not used in the analyses.

Laboratory analyses on the dike embankment and foundation soils included soil classification (Atterberg limits, specific gravity, sieve and hydrometer analyses), and shear strength (triaxial compression with pore pressure measurements, and unconfined compressive strength) tests. Additionally, the laboratory analyses contained in the previous geotechnical studies were reviewed and those test results that were judged to be reasonable were included in the data sets for slope stability analyses.

Figure 7 is a plan view of Ash Disposal No. 2 showing the locations of borings drilled during this study, and of borings drilled during the six previous studies.
**Figure 11: Boring Locations**

**Results of Geotechnical Exploration:** Based on the results of the drilling, laboratory testing, historical documentation, and drawings, the dike and foundation soils at Ash Disposal Area No. 2 were analyzed as five soil layers. These layers shown in Figure 5 and are briefly described in ascending order of lithology as follows:

- **Alluvial Sand and Gravel:** This represents an alluvial sand and gravel layer that directly overlies bedrock (estimated to be at approximate Elevation 290 feet msl). It is medium dense to dense in relative density, and extends upwardly from bedrock to Elevation 320 - 335 feet msl.

- **Alluvial Clay and Silt:** This layer overlies the alluvial sand and gravel. Strength consistencies are very soft to medium. It is from 15 to 20 feet in thickness and extends to Elevation 340 – 355 feet msl.

- **Hydraulic Fill:** This layer consists primarily of low plastic clay or silt intermixed with sandy and gravelly zones. Consistencies range from very soft to medium. Outslopes are variable, relatively flat, and extends up to Elevation 365 - 370 feet msl.

- **Lower Clay Dike:** This is the first dike placement that employed compacted clay. It was constructed to a crest Elevation 378 feet msl. It consists primarily of medium to very stiff low plastic clay.

- **Upper Clay Dike:** This is the second dike placement that raised the dike to its current crest at Elevation 390 feet. It consists primarily of stiff to very stiff low plastic clay.
Figure 12: Typical Dike Cross-Section

Geotechnical Engineering Analyses: Nine cross-sections were selected for seepage and slope stability analyses. Selections were based on dike geometry, phreatic surfaces measured within dikes, and shear strengths determined from laboratory and field testing. Seepage and slope stability analyses were performed and the long term steady state seepage factors of safety for both the seepage and slope stability analyses were calculated at each cross section. The following describes the general procedures that were used to determine the factors of safety.

An analysis of steady state seepage through the dike was first performed to estimate the magnitude of seepage gradients (for the evaluation of potential piping) and pore water pressures within the soils (for the evaluation of slope stability). The numerical seepage models were developed using SEEP/W 2007 (Version 7.15), a finite element code tailored for modeling groundwater seepage in soil and rock. SEEP/W is distributed by GEO-SLOPE International, Ltd, of Calgary, Alberta, Canada (www.geoslope.com).

Boundary conditions for the SEEP/W analysis were assumed as follows. Along the vertical, downstream edge of the model, the hydraulic head at each boundary node was constant with depth and assigned a value equal to the summer Kentucky Lake pool elevation. The vertical, upstream edge of the model is located along the longitudinal center-line of Ash Disposal Area No. 2 and modeled as a no-flow boundary (i.e., Q=0). The basis for this assumption is that the pond water would take the shortest path to the perimeter dike, and that the center-line is the dividing line for the direction of flow, hence a no flow boundary is considered appropriate. A total head value equal to the pool level was applied to all submerged nodes along the ground surface of the upstream side (submerged sluiced ash and interior upper dike). Other nodes along the ground surface were treated as potential seepage exits. At various steps in the computer analysis, if
the software determined that water flows from the mesh at these nodes along the ground surface, SEEP/W assigned a head equal to the elevation of the node. This routine effectively models the seepage exit to the ground surface. The horizontal boundary at the base of the model (bedrock surface) was modeled as a seepage barrier, with no vertical flow across the boundary nodes. For each modeled cross-section, a representative subsurface profile was compiled based on boring logs, available record drawings, and the known project history. Material properties were estimated based on available laboratory data, correlations with classification data, and on typical values for similar materials.

Significant engineering judgment is needed to select appropriate hydraulic properties for earth/soil materials. Unlike other key properties, hydraulic conductivity can vary over several orders of magnitude for a range of soils, often with substantial anisotropy for seepage in horizontal versus vertical directions. Laboratory test samples often do not represent important variations within a larger soil deposit. An iterative process of parametric calibration was used to arrive at final estimates of the seepage properties. Results from trial simulations were compared to field data (measured piezometric levels and observed seepage) and the material parameters were then varied until the solutions reasonably matched the field data. The ratio of horizontal hydraulic conductivity \((k_h)\) to vertical hydraulic conductivity \((k_v)\) was estimated based on placement, depositional characteristics, and origin of the materials. An isotropic material would have \(k_h/k_v = 1\), while deposits of horizontally layered soils will have much higher values. In general, higher ratios were used for alluvial soil deposits, than for the compacted dike materials.

After the initial seepage parameters were estimated, results from the SEEP/W model were compared to groundwater levels measured in piezometers installed within the borings. Nodes were placed in the model at the same location as the piezometer tip was installed in the field so that the total head predicted at the node could be compared to the corresponding piezometer reading. The material properties in each modeled cross-section were then varied until a reasonable match was obtained between the seepage predictions and field data. Specifically, the saturated hydraulic conductivity and the \(k_h/k_v\) ratios were adjusted (while still maintaining the parameters within expected ranges) to give model predictions as consistent as possible with field measurements and observations.

The comparison between the field piezometer measurements and final SEEP/W predictions showed the predicted groundwater table ranging from about 1 foot below to 7 feet above the readings obtained in the piezometers installed within the dike crest. For the dike toe areas, the seepage model consistently predicted the water table position to be from 3 feet below to 3 feet above actual toe piezometer readings. The results from the seepage model can also be compared to field observations of seepage. For Ash Disposal Area No. 2, historical seepage has been present along the majority of the northeast and southeast dikes (see Figure 9). These observations correlated well with the seepage models for the cross-sections which generally show the shape of the predicted phreatic surface extending to the slope face.
Figure 13: Historical Seepage Locations at Ash Disposal Area No. 2
The slope stability analyses were performed using SLOPE/W 2007 on the exterior dike slopes. SLOPE/W incorporates various search routines to locate the critical slip surface. For these analyses the "Entrance and Exit" method was employed. The distribution of pore water pressures obtained from the SEEP/W model was also used.

The lower dike at Ash Disposal Area No. 2 was originally constructed in 1970, and the upper dike in 1978. The dike has existed in its current cross sectional geometry (slopes and crest elevation) for at least 40 years. Hence, excess pore pressures generated in the underlying soil during construction have had sufficient time to dissipate and steady state seepage conditions have developed. The stability analyses addressed static steady state seepage conditions, and drained strength parameters are needed.

The drained shear strength parameters were derived using results of laboratory triaxial tests, along with consideration given to standard penetration test data and laboratory classification test data. In addition, the strength parameters selected were further refined or confirmed by comparisons with the strength parameters listed in the previous geotechnical reports. Representative strengths for each horizon were selected using methodology outlined in the US Army Corps of Engineers Engineer Manual EM 1110-2-1902 as a guide. Results of triaxial testing were evaluated and effective stress $p'$ versus $q$ scatter plots were prepared of all of the data points. The maximum effective principal stress ratio was used to determine failure criteria for selection of these values. Once a $p'$ versus $q$ plot was prepared, a failure envelope was then selected such that (minimum) two thirds of the plotted values were on or above the envelope. An example $p'$ versus $q$ plots and selection of the failure envelope is shown in Figure 10.

![Figure 14: Graph Used to Determine Shear Strength](image-url)
For non-cohesive alluvial sands and gravels, shear strength parameters were estimated using published relationships which correlate SPT N-values with relative density, specific soil types and angles of internal friction. In addition to the dike and foundation soils, sluiced ash materials are present below the upstream toe of the upper dike. Shear strength parameters for ash materials were estimated using historical data, typical values, and published correlations using SPT N-values.

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Results of Slope Stability Analyses: Using these strength parameters the existing dike configurations were analyzed at the nine selected cross-sections. Geo-Slope’s SLOPE/W computer program was used for the analyses with pore pressures imported from the seepage analyses. Long term (effective stress) steady state seepage conditions were analyzed using Spencer’s method. For the Spencer’s method analyses, circular failure surfaces with optimization were conducted. Figure 11 shows a graphical view of a typical SLOPE/W output file, and Figure 12 depicts the minimum factors of safety calculated at for deep-seated (global) failure surfaces.

![Figure 15: Typical SLOPE/W Output File](image-url)
Figure 16: ‘As-Found’ Factors of Safety

There was no indication in the slope stability analyses that a noncircular failure surface would give a factor of safety lower than that obtained for circular surfaces. Overall, the geometry of the dike cross-sections and the foundation stratigraphy do not appear to be susceptible to sliding along a planar surface. The optimization scheme available within Slope/W was used to consider noncircular, curved slip surfaces.

The guidelines presented in USACE Manual EM 1110-2-1902 “Slope Stability” and in accordance with current prevailing geotechnical practice, indicate that a minimum target factor of safety of 1.5 is appropriate for long-term, steady state seepage conditions. The results of the analyses reveal that the factors of safety range from 1.2 to 1.7, but that only four cross-sections (Sections A, I, K, and M) have safety factors that meet the target value. The lowest values were obtained along the Northeast dike, and the next lowest were along the Southeast dike. Analyses on the West dike exhibited factors of safety for deep-seated global failure surfaces that met the target.

The final report for of the Geotechnical Exploration and Slope Stability Evaluation for Ash Disposal Area No. 2 was issued in April, 2010. It is noteworthy that the geotechnical report has undergone three separate peer reviews by different geotechnical firms, with each having demonstrated expertise in dams, tailings dams and CCP disposal facilities. In each case the independent experts did not find any reason to challenge the scope or results of TVA’s work investigating Ash Disposal Area No. 2.

While the geotechnical and slope stability evaluation was performed, TVA made the decision to close its wet CCP impoundments, including Ash Disposal Area No. 2. In December, 2009 a presentation titled “Johnsonville Active Ash Disposal Area Closure Plan” was provided by TVA to the Tennessee Department of Environment and Conservation (TDEC). This plan described interim risk reduction projects that will be
implemented by end of FY 2011. Each project will reduce risk and provide a minimum
long-term factor of safety equal to or greater than 1.5. The projects having the greatest
impact on risk reduction are in these two categories:

- **Spillway Replacement and Closure Projects** reduce risk by constructing new,
safer spillways and lowering the ash pond complex pool levels. The added
freeboard reduces overtopping risk. The stacked RCP riser spillways that are
leaning, and pipes that have documented history of damage, are removed
from service. The lowered pool level reduces seepage, allows the sluice
channel to be lowered, and results in lowered phreatic surfaces and higher
stability factors of safety throughout the facility. Last, it sets the stage for
outlet pipe closures, and eliminates the risk associated with open pipe
penetrations through the dikes.

- **Northeast and Southeast Dikes Slope Improvements** reduce risk by improving
the slope stability and piping (internal erosion) factors of safety. This is done
by constructing rock buttresses to stabilize steep slopes below the lower
bench, installing graded filters at documented seepage areas, flattening
exterior slopes with compacted clay, and improving accessibility for inspection
and maintenance by constructing all-weather access roads at the dike toe.

To implement these risk reduction projects a Johnsonville design team was developed.
The team included managers, engineers, environmental and other staff from TVA's
Fossil Power Group (CCP Projects & Engineering, Environmental Permitting &
Compliance, and Routine Handling & Maintenance), the Johnsonville Fossil plant, and
Stantec Consulting Services.

**SPILLWAY REPLACEMENT AND CLOSURE PROJECTS**

**General Information:** These projects were initiated at the same time the geotechnical
exploration was beginning in February, 2009. Recognizing that the existing freeboard
was inadequate and that the existing spillways posed risks, TVA decided to install new
spillways and take the existing spillways out of service as soon as possible. Later, the
old spillways would be closed after a period that allowed investigation of potential
damage, and development of a safe closure plan that considered risks and
contingencies.

It was decided that the new spillways should be located in the same general vicinity as
the existing outlets, so that the NPDES permit would remain unaltered. The design was
completed and plans were “Issued for Construction” on May 15, 2009. It is noted that
these plans were developed for the first spillway replacement project TVA would
implement at its CCP facilities; therefore, future project design teams would “learn” from
this one.

The existing spillways were built in three sets, with each set consisting of three
riser/outlet pipes (total of nine spillways). They are designated the South, North and
East spillways. The South and North spillways are on the west side of Ash Disposal
Area No. 2, and these discharged directly to Kentucky Lake. The East spillways are through the Southeast dike, and they discharged to the Condenser Water Inlet Channel. The North and South outlet pipes are about 200 to 250 feet long. The East pipes extend beneath a constructed peninsula and are about 700 feet long. As mentioned earlier the pipes are 36-inch diameter RCP. Refer to Figure 2 for the general locations of the existing spillways.

**Spillway Replacement Design Considerations:** Considering the environmental consequences in event of failure, Ash Disposal Area No. 2 is a Category 2 (significant hazard) structure in accordance with rules applied to the Tennessee Safe Dams Act of 1973. Based on height and storage, the facility is classified as Intermediate in size, and the Freeboard Design Storm is required to be one half the 6-hour Probable Maximum Precipitation (PMP) event. However, TVA decided to design the new spillways for sufficient capacity to safely pass the entire 6-hour Probable Maximum Precipitation storm without overtopping the dike. The 6-hour PMP event for Humphreys County, Tennessee is about 35 inches of rainfall. A one foot freeboard was added to the maximum water surface elevation to account for wave run-up. These criteria set the new spillway crest at Elevation 384.0 feet, which is 2.5 feet lower than the existing spillway crest.

Other design criteria included: capability to vary the pool level in the ash pond complex, use leak-proof pipe joints, provide ability to inspect outlet pipes, prevent settlements, internal erosion control, retention of cenospheres, and provide emergency drawdown capability. With these criteria in mind, the TVA design team considered various spillway types and ended up with the following design.

The new spillway system consists of six precast-concrete inlet structures set side-by-side. The structures are benched into the existing dike to reduce net total weight, and eliminate settlement potential. The pool is controlled by 7-foot long fiberglass stop logs. The stop logs are six inches in height, can be easily added or removed, and are gasketed to reduce leakage. A corrugated steel skimmer surrounds the structures so that cenospheres cannot escape from the pond. Each inlet structure is drained by a 30-inch diameter High Density Polyethylene Pipe (HDPE) that passes beneath the dike crest at a shallow grade, and then down the slope at a steep grade. The pipe is vented where the slope increases to supercritical. A seepage filter around the outlet pipes controls seepage and prevents internal erosion of the clay dike. An outlet pipe headwall, which includes a raised sill for energy dissipation, is at the downstream end above the Kentucky Lake summer pool level. The following figures show the inlet and outlet ends of the new spillway system.
Figure 17: Inlets for New Spillways During Construction

Figure 18: Outlet End of New Spillways
A siphon system was designed to draw down the pool during construction, and provide emergency drawdown. The design team determined that the system needed sufficient capacity to pull the pool down in 10 days, plus the plant inflow of 32 mgd. To prevent cavitation, the maximum flow rate needed to be limited to 15 feet per second (fps). Last, the inlet velocity had to be low to prevent ash from being pulled into the siphon, especially when the pool reached the maximum drawdown limit. The team decided that the inlet velocity should be less than 1.5 fps, and velocity vectors should generally be from the side or downward (away from settled ash).

With these parameters, the team’s design consists of the following. The siphons are four 18-inch diameter HDPE pipes, each with a 34 foot long “torpedo” inlet. A series of 4-inch diameter holes over the inlet length diffused water entering the siphon and lowered the inlet velocity. The top of inlets are maintained approximately 4 feet below the pond surface and are suspended from 30-inch diameter HDPE-covered foam float balls.

A priming port and valve is at the crest. Each outlet is equipped with a knife gate valve, to adjust flow, and 45 degree bend to deflect the discharge upwards into the air. Large rip-rap in the landing zone prevents scour of the lake bank. Figures 15 and 16 show the torpedo inlet with floats, and the siphons discharging at full flow during pool drawdown.

**Figure 19: Siphon Inlet with Float Balls**
Figure 20: Siphons during Pool Drawdown

Spillway Replacement Construction: TVA’s Environment and Technology group (formerly Office of Environment and Research) from Muscle Shoals AL was awarded the contract to construct the spillway replacements (and also the spillway closures). Following the issuance of required environmental permits from the US Army Corps of Engineers and the Tennessee Department of Environment and Conservation Work began in early July, 2009, and was complete in early November. Figure 17 is an aerial photograph taken during the mid-point of construction.
Figure 21: Spillway Replacement Project

The project went smoothly, and there were few issues during the 82-day construction period. Because this was the first spillway replacement project to be undertaken by TVA, “lessons-learned” were documented as construction progressed. Most were related to minor design changes, and a lessons-learned presentation was given to design teams on similar projects at TVA’s Cumberland, Colbert, and Shawnee fossil plants. The most noteworthy items are:

- Cast-in-place concrete structures would have been better-suited for the work than pre-cast structures. This is because elevations and tolerances can be adjusted easier to fit the field conditions, lifting equipment can be lighter and pre-cast plant delivery hold-ups and quality issues can be avoided.

- The design needed to anticipate additional erosion and head cutting in channels during pond draw down and include provisions for structural controls as needed.

- The design needed to anticipate the effect of pool drawdown on the pH control system. In this case the system could not operate effectively and acid deliveries were required in order to maintain compliance with the NPDES permit.

At project completion the ash pond complex was lowered from normal pool level 387.5 feet msl to 384.6. This provides an additional 2.9 feet of freeboard, or 5.4 feet total, an amount sufficient so the dike will not be overtopped during a PMP storm event. The
freeboard also now meets TVA’s Master Programmatic Documents, which specifies a minimum of 5 feet at CCP impoundments. Last, the lower water level has reduced seepage pressures throughout the dike system. In particular, the dry weather base flow measured at the Seep 3A collection system outfall dropped by approximately one third, from 3500 gpm to 2300 gpm.

**Spillway Closure Design Considerations:** This project followed the Spillway Replacement, and TVA Fossil Group Management desired that it be completed without delays due to Kentucky Lake operations. Therefore it was decided that the closure plan would be designed to allow work when Kentucky Lake is at maximum (summer) pool. Other design considerations were: pipes must be cleaned, TV-inspected and video recorded, a graded filter must be installed around the end of each pipe, and a contingency plan against ash release during “the critical period” must be implemented. The grout mix must retain fluidity during pumping, be free from shrinkage, and develop enough short term strength for early form removal. Last, grout would be pumped until the grout level measured in the riser rose to ten feet above the invert, or the pressure recorded at the bulkhead reached 25 psi.

Using these criteria, the design team developed the following plan. Steel sheet pile cofferdams would be used to retain excavation slopes and provide seepage control. Based on the excavation depth, soils, and hydrostatic head, the piles would need to be extending about 25 feet below grade. PZ 22 sections would have satisfied the earth pressure calculations; however, PZ 27 sections, using Grade 50 steel, were selected for extra durability. It was planned that the piles be pulled and reused, not only at JOF but at other TVA plants where spillway closures were planned. The cofferdams were to be 22 feet square in plan and one line of wales was needed.

The team also designed a steel bulkhead equipped with grout ports, valves and pressure gauges, and the struts and screw jacks to hold the bulkhead securely against the outlet pipe (see Figure 18 below). The bulkhead was made from 5/8-inch thick steel plate. Four-inch diameter Schedule 40 black steel was used for grout pipes, and bronze ball valves were used to block flow at completion of grouting. Three bulkheads were fabricated and each was reused twice during this project. It is intended that they be reused on other TVA pipe closure projects.
As part of the contingency plan, an inflatable plug would be inserted into the riser base at the beginning of a defined "critical period". The design team worked with the local ready mix company to develop a mix design. After testing several trial batches, the team selected mix a mix design with the following proportions (per Cu. Yd.):

- 525 lbs Cement
- 130 lbs Fly Ash
- 1850 lbs Concrete Sand
- 1000 lbs Pea Gravel
- 32 oz High Range Water Reducer (Glenium 7500)
- 20 oz Set Retarding Admixture (Pozzolith 300R)
- 40 gal Water

**Spillway Closure Construction:** TVA’s Environment and Technology group began construction in May, 2010 and completed the work in February, 2011. The project started at the East Spillways because it was believed that risks were lower due to the East site being located on land, rather than in Kentucky Lake. If problems developed, solutions could be worked out prior to moving to the South and North spillway sites.

Unfortunately, problems developed during the sheet pile driving. Although exploratory borings had been advanced during the design, it became apparent that various types of debris had been buried when the spillway pipes were installed during 1978. The sheet piling occasionally encountered buried trees, steel cables, and other obstacles that
slowed the driving operations and made it difficult to hold proper alignment. The Contractor experienced delays caused by equipment damage or replacement, and while misaligned piles were pulled and re-driven.

After the cofferdams were installed, the pipes were excavated and then opened up. The cleaning and CCTV inspection revealed the outlet pipes to be in good repair. The grout bulkheads, struts and backstops were installed, and the grouting operation began. Approximately 60 cubic yards were pumped into each pipe and the period for actual grout placement was about 60 to 90 minutes per pipe. Graded filters were then installed, piling pulled, and the site was graded and seeded. Figure 19 provides a view of the site during the grouting operation.

Figure 23: Grouting the East Spillway Pipes

In October, 2010 the Contractor moved closure operations to the South and North Spillways. By this time the reservoir level at Kentucky Lake had already been lowered to winter pool. Since the water level was lower, the Contractor proposed an alternate cofferdam plan in lieu of driving sheet piles. The proposed plan would use rock (small rip rap and crushed stone) to build a work platform around each pipe outlet. Polyethylene (PE) sheeting would be placed around the crushed stone for seepage control.

Following a review by the design team, the proposed plan was modified to change the PE sheeting to 20 mil PE pond liners. The pond liner was also specified cover the lake bottom a distance of 25 feet beyond the crushed stone slope. Last, additional seepage
analyses were performed, and an estimate of required pumping capacity was provided to the Contractor. Using this information, the Contractor brought larger capacity pumps to the site, and was successful in pumping down and maintaining dewatered work areas within the rock cofferdam.

Figure 24: Grouting the South Spillway Pipes

Similar to the East spillway site, the cleaning, TV inspections and grouting operations at the South and North spillway sites went smoothly and completed without incident. The outlet pipes at two of the North spillways could not be cleaned and grouted completely due to a very hard concrete-type residue that blocked the pipes. This condition was due to a previous TVA effort to close these spillways by filling the risers with concrete, and apparently some of the concrete partially filled these two outlet pipe. However, the measured grout take during this project indicated that the two pipes were both filled to the obstructions. Therefore, no additional work was considered necessary. The outlet pipes closures were completed in February, 2011 and remaining punch list items were finished in March.

NORTHEAST & SOUTHEAST DIKE STABILITY IMPROVEMENTS

General Information: These two projects involved constructing rock toe buttresses, graded filters and placing compacted clay to flatten exterior dike slopes. These construction activities will increase the slope stability factors of safety to greater than 1.5, and address seepage areas throughout the entire east side and south end of Ash Disposal Area No. 2. Another risk reduction benefit is that the exterior slopes will be
safer to mow and maintain, and improved access with all-weather rock roads will make inspections safer and easier. The total length involved is 5,000 feet, or about one half the distances around the dike.

TVA decided to do the work as two separate projects, as it was felt that since the worksites are separated by the Causeway, they can be managed and inspected easier as two projects. Recognizing that the Northeast section of the containment dike exhibited the steepest slopes and lowest stability factors of safety, TVA decided to mitigate it first. Due to the estimated truck traffic that would be developed during rock and soil deliveries, it was determined that the single causeway road was not sufficiently wide to run both projects concurrently and safely.

**Figure 25: Northeast Dike Prior to Slope Improvements**

Before the work would be started it was decided that sluicing channel with its associated ash dipping, should be relocated away from the dike. The old sluice channel could then be dewatered and the construction project could proceed with some of the risk already reduced. This work was performed by TVA’s ash handling contractor and it was completed in early 2010.

**Design Considerations:** Figure 26 shows a typical cross section from the Northeast Dike plans. It depicts most of the design considerations for this project as well as the Southeast Dike project. Beginning at the toe, they include:
• Riprap starter berm to define the outer limits of the rock buttress and ensure subsequent fill placement is within the area prescribed by an agreement between TVA and US Fish and Wildlife for protection of a threatened and endangered mussel.

• Graded filter consisting of coarse aggregates and sand constructed beneath the rock toe buttress and up the lower dike slope. The filter allows seepage to continue, but prevents internal erosion of dike or foundation soils. Filters are designed in accordance with US Army Corps of Engineers EM 1110-2-2300 “Design and Construction Considerations for Earth and Rock-Fill Dams”

• Riprap buttress against the lowest slope where the dike system grades down below the Kentucky Lake reservoir water level. The buttress is designed to dimensions that provide a counterweight at the toe to resist global, deep-seated slope failures and prevent shallow maintenance-type failures of the lower slope.

• Crushed stone drainage blanket is a filter and seepage conveyance of sand, gravel, and geotextile fabric. Its purpose is to provide drainage for seepage areas that currently exist at the base of the upper dike.

• Shallow serrated slope benches cut into the dike as the compacted soil fill is built upwards. These benches eliminate the potential slip plane and provide horizontal surface for compaction.

• Compacted Soil Fill flattens the exterior dike slope, improves global slope stability and stability of shallow, surficial sloughs.

Figure 26: Cross-Section from Northeast Dike Plans
Two other design considerations were addressed by the design team. The first was an assessment of the lower dike for its stability under the anticipated equipment loads (this was requested by the Contractor following the Pre-Construction meeting). The second was requested by JOF Yard Operations near the completion of construction. It involved a redesign of the slope to include a gabion retaining wall that would provide additional clearance for a crane to operate in the area of JOF’s work barge dock.

![Figure 27: Factors of Safety after Slope Improvements to the Northeast and Southeast Dikes](image)

**Figure 27: Factors of Safety after Slope Improvements to the Northeast and Southeast Dikes**

**Construction:** TVA’s Clean Strategies and Project Development - Civil Projects Group was contracted to build both the Northeast and Southeast Dike projects. Construction of the Northeast Dike project began in late February, 2010, following a pre-construction meeting on February 18. The project was completed in August, and a final walk-down punch list meeting was held at the site on August 24. There were 103 construction work days over the 177 calendar day period. Full-time construction observation and quality control testing services were supplied by the design team. The project went smoothly and there was only one weather-related work delay. This occurred in early May when over 17 inches of rain fell causing record flooding in central Tennessee, and the Kentucky Lake level rose to Elevation 370 feet.

There were two unforeseen incidents during construction that required action by the design team. The first occurred in early March during the initial stage of rock buttress construction. The riprap buttress was about 15 feet in total height and a short section, approximately 300 feet north from the causeway and 50 feet in length, settled suddenly creating 1.5 foot vertical scarp. The scarp appeared to be located near the original bank slope. This indicated a localized bearing capacity failure beneath the toe had allowed the short section of riprap to slip downward along the original slope. After
several days of survey monitoring the area stabilized and additional rock was placed to bring the area to grade. No further movement was recorded in this area. Based on this occurrence, the Contractor began limiting lift thicknesses to allow more time for underlying materials to adjust to the added rock weight.

Later that month, a second slip occurred over a 100 foot length of the buttress approximately 1500 feet north of the first slip. Based on its geometry, it also appeared to be caused by a bearing failure beneath the buttress toe and movement along the approximate original bank slope. Its movement stabilized over a longer period of several months. Since the underlying material appeared to be weaker than at the first slip, the design team decided to eliminate the scarp by grading rock back, rather than adding additional rock to bring it up to the design grade. It was also necessary to inform the US Fish and Wildlife Service of the movement, because a short section was determined to have moved a few feet beyond the limit established by TVA and USFW.

Both slips were studied by the design team and neither was considered serious or adding risk. However, they underscored the need to ensure that geotechnical borings are advanced during design, even in areas accessible only by a work barge. Borings were not advanced in the Boat Harbor during the design.

![Figure 28: The Northeast Dike Project Nearing Completion](image)

In summary, the Northeast dike project involved approximately 2500 linear feet of dike improvement. The rock buttress eliminated the steep bank slope and extended the
lower bench from 15 to 20 feet towards the Boat Harbor channel. The clay dike was flattened from its original 1.5H:1V slope to 2.5H:1V. About 30,000 tons of riprap, 5,000 tons graded filter (sand and crushed stone), and 14,000 cu.yds of clay were used in the work. The Contractor elected to lay sod in order to achieve immediate stabilization of the site.

Figure 29: Northeast Dike after Completion

Civil Project Group moved its forces directly from the Northeast Dike and began work on the Southeast Dike project in September, 2010. The original schedule indicated work would be complete in early March; however, the weather between December and March was extremely wet and many days were lost due to wet conditions. At this writing, the current schedule indicates a completion date in early June, 2011.

The work performed on the Southeast Dike is similar to the Northeast Dike. One notable difference is Seep 3A. The collection system was removed and the area was stabilized with a biaxial geogrid, prior to placing the graded filter. In one localized area it was necessary to undercut and replace with riprap, prior to placing the geogrid and graded filter.

To date, one incident has occurred that required an action by the design team. During the tree clearing operation on the lower bank near Seep 3A, the Contractor encountered increased seepage along a 50 foot long section of the lower bank. This was mitigated by stabilizing using riprap, and then constructing a substantial graded filter. Geogrids
were also used to strengthen the toe area, and the rock buttress was completed as designed. The design team also recommended leaving roots and root wads in-place throughout the more pervious zone in the old hydraulically placed fill.

![Image of construction site](image.jpg)

**Figures 30: Graded Filter at Seep 3A**

**SUMMARY AND CONCLUSIONS**

This paper has described the risk reduction program developed for Johnsonville Fossil Plant Ash Disposal Area No. 2 following the Kingston dredge cell failure of December, 2008. A Phase 1 Assessment for the facility resulted in a high priority ranking for risk reduction actions. The assessment uncovered issues of concern such as inadequate freeboard, spillways and pipes, steep slopes, seepage, and construction techniques not appropriate for a water-impounding structure. Although the assessment did not determine that there was an urgent and compelling need to take immediate corrective action, the issues of concern were considered serious enough that TVA moved quickly.

As soon as the initial Phase 1 findings were available TVA began a risk reduction program at Ash Disposal Area No. 2. In a very systematic way, it solved the spillway and freeboard issue first by designing and constructing a new spillway system, which also included siphons that can be activated for emergency drawdown. With the old spillways taken out of service, TVA was able to reduce the risks associated with open pipe penetrations by constructing filters and filling all of the old outlet pipes with grout.
In the meantime a comprehensive geotechnical exploration was performed and the results of geotechnical analyses were used to quantify the level of risk at each section of the dike system. The information was also used to design risk reduction projects with the purpose of increasing slope stability and seepage factors of safety to accepted criteria for dams. This work will be completed in June, 2011.

Last, TVA also made changes in the maintenance and ash handling operations at Ash Disposal Area No. 2. Although not described earlier, these include increased mowing and vegetation removal, elimination of animal burrows, removal of significant ash stored at the facility, and relocating the sluice channel and ash dipping operation to be away from the containment dike. While difficult to quantify, the modified operations and maintenance activities have had a very positive impact on risk reduction.

All of the actions that comprise the risk reduction program have taken place while Ash Disposal Area No. 2 remained in full operation and handling all of the CCPs produced at the plant. The Johnsonville Design Team and Contractors will meet TVA’s goal for completion by mid-2011. At that time Ash Disposal Area No. 2 will have achieved the Tennessee Safe Dam Program requirements for an existing “significant hazard structure.”

REFERENCES


